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# The influence of tensile membrane action on fire-exposed composite concrete floor-steel beams with web-openings

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**Abstract**

On the basis of a series of the full-scale fire tests carried out on the composite frame at Cardington, a design method is now well established to calculate the performance of composite flooring systems subject to fire. The method models in a simplified fashion the influence of tensile membrane action in composite floor slabs which are formed as an array of composite beams using solid-web steel downstand beams which are largely unprotected. The development of membrane action depends on the conditions of vertical support maintained around the boundaries of the fire-affected slab panels by protected beams. Cellular beams can achieve the same strength as unperforated I-beams of the same depth, with significantly reduced steel mass, and the ability to accommodate service ducts within the beam depth, and therefore the use of composite cellular beams as floor members is becoming increasingly popular in construction. Due to the general lack of research on perforated sections, the guidance about their design for fire conditions remains rather primitive. Moreover, tensile membrane action in composite floor slabs with cellular steel downstand beams could exhibit very different behaviour in fire from that when solid-web sections are used. A parametric study has been carried out as an initial investigation into the fire performance of cellular beams within composite slab systems, including the effects of tensile membrane action in enhancing the load-carrying capacity. The effects of changes to the edge support conditions are also investigated. The results show the protected perimeter beams maintained their load-carrying capacity and were subject only to small vertical displacements, although web-post buckling was observed to occur on the protected secondary beam near to a support. This suggests that maintenance of vertical support is not sufficient for a slab panel with cellular beams. Web-post buckling or the Vierendeel mechanism may govern the mode of structural failure, indicating that the sizes of openings and their positioning necessitate careful design.


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*Keywords*: Composite beams; Web-openings; Finite element modelling; Tensile membrane action

## 1. Introduction

The traditional approach for structural fire engineering of steel-framed buildings is simply to apply fire protection to all exposed structural members to ensure that they retain their strength and stiffness during a fire. Simplified design methods [1, 2] assume that individual structural elements behave independently in fire, ignoring interactions within the composite system. Observations of how framed composite steel-concrete structures behave in fire, taken from the full-scale fire tests on multi-storey building which were conducted at the BRE Fire Research Laboratory at Cardington UK [3, 4], have shown that a real structure will behave much better in a fire than its individual elements. This happens because, when a part of the structure is subjected to high temperature, with rapid loss of stiffness and strength, the redundant structure often finds alternative load paths. Large deflections develop, which contribute to the survival of the entire structure by permitting the

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development of tensile membrane action in slabs and catenary action in beams. Tensile membrane action of slabs is a geometrically non-linear behaviour which relies mainly on the conditions of vertical support which can be maintained around the boundaries of individual slab panels, but is also influenced by the restraint to in-plane edge movement imposed by surrounding structure. The same principles control the development of tensile membrane action in a slab panel surrounded by solid-web perimeter beams and for one surrounded by beams with web-openings. However, the use of steel beams with web-openings as the downstands of a composite floor leads to questions being asked about how additional potential failure modes, particularly at elevated temperatures, influence the vertical support along the edge of the slab panel. It is important that this influence is investigated and fully understood, given that such arrangements are increasingly used in practice.

This paper presents a parametric study which has been carried out as an initial investigation into the fire behaviour of cellular beams within composite slab systems. The current study of the influence on high-temperature tensile membrane action in slabs of the presence of web-openings in beams is based on a finite element approach developed and validated by Wong et al. [5]. It has examined the ways in which the BRE membrane action method is affected by the degree of vertical support available from composite cellular beams along the slab panel boundaries.

## 2. Composite floor beams with regular web-openings

Structural steel beams with regular web-openings also well known as cellular beams, are the modern version of the traditional castellated beams. The use of composite steel beams with regular web-openings is becoming increasingly popular in multi-storey building construction. The system is capable of providing passage for building services through the openings in the steel web, thus reducing the height of the overall floor zone. In practice, web-openings in beams result in different stress distributions within the webs compared to solid-web sections, and these can create unique failure modes. A web-opening causes a Vierendeel-girder action over its length due to the possibility of relative shear displacement between its ends. This can result in the formation of four plastic hinges at the "corners" of the opening (Fig. 1(a)). Horizontal shear forces transferred across the web-posts between openings can result in local buckling of these web-posts (Fig. 1(b)). In fire, the degradations of strength and stiffness of different parts of an unprotected section happen at different rates, and this can cause not only early structural collapse but also a change of critical failure mechanism compared to ambient-temperature performance.

Current UK design procedure for composite floor beam with web-openings is based on Steel Construction Institute (SCI) publications [6, 7], as well as theoretical guidelines based on previous research studies [8-11]. These provide design guidance for composite and non-composite cellular beams, particularly at ambient temperature. Recent enhancements in understanding of cellular beams in fire have been made through a number of full-scale fire tests in various European research projects [12, 13] and finite element modelling [5, 13]. However, these have been limited to fire tests on isolated beams, which are unrealistic.

## 3. Tensile membrane action method

Following the full-scale fire tests on a steel-framed office building at Cardington [3, 4, 14, 15] a membrane action design method for composite concrete floors subjected to fire was developed. The composite floor is divided into slab panels, supported on protected edges which withstand vertical deflection, and incorporating internal unprotected composite beams as shown in Fig. 2. Exposure to high temperatures causes the unprotected steel beams to lose strength and stiffness rapidly. Plastic hinges form in the unprotected beams, resulting in a redistribution of their loads via two-way bending of the slab as it undergoes large vertical deflections. The method assumes that the tensile membrane action mechanism which occurs at ambient temperature is maintained at elevated temperatures [14], and that the protected boundary beams remain vertically undeflected throughout a fire. Research has, however, shown [16, 17] that the development of tensile membrane action at elevated temperatures differs from that at ambient temperature, in that the protected perimeter beams lose strength and stiffness, at some point allowing the formation of a single-curvature slab-bending mechanism, which can lead to a catastrophic failure of the structure. In recent years it has become common engineering practice to use beams with web-openings, applying the principles of membrane action of composite floor slabs with solid-web steel beams in fire, and hence to omit fire protection from secondary beams. In practice, slab panel support is realised by protecting the perimeter beams. The assumption of continuous vertical support at all times during the fire is clearly unrealistic; moreover, tensile membrane action in composite floor slabs with cellular downstand beams could exhibit very different behaviour from that with solid-web sections in fire, due to the failure modes which are unique to these beams. This has prompted the finite element studies reported here, into the adequacy of vertical support along the slab panel boundaries.

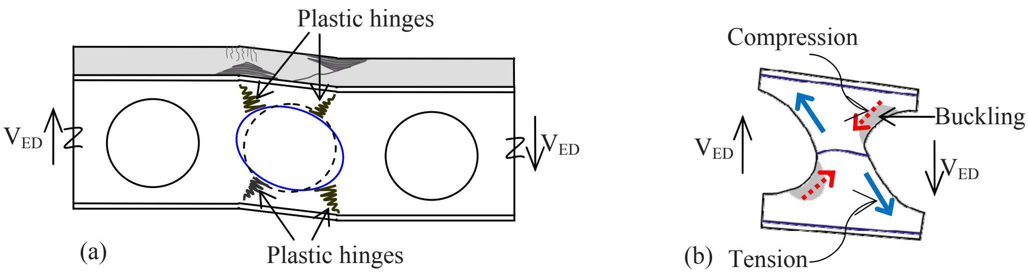


Fig. 1. Mode of failure (a) vierendeel mechanism and (b) web-post buckling.

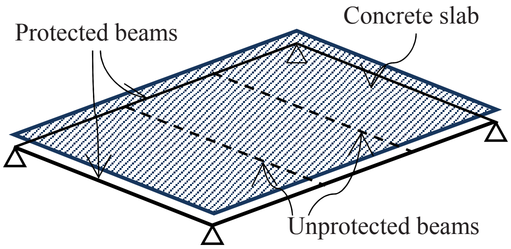


Fig. 2. Slab panel, showing protected and unprotected beams.

## 4. Slab panel failure study

Based on a previously developed and validated finite element model [5], parametric study has been performed to investigate the BRE membrane action method using downstand beams with regular web-openings. A 9 m x 12 m slab panel was used in the study. The panel has two unprotected beams spanning in the long direction, while the perimeter beams are protected, as shown in Fig. 3. Ambient and elevated-temperature design of the floor beams, based on SCI [18] and the previously developed design methods by the author [19], resulted in all secondary beams being formed from 356 × 171 × 57UB and primary beams from 406 × 178 × 74UB. All beams had a final overall depth of 650mm, with 400mm diameter openings in the webs, as shown in Fig. 4. Composite slabs of 130mm thickness with a steel decking profile depth of 51mm were specified. The compressive strength of the concrete slab was 35 N/mm$^2$, with a layer of A252 reinforcement mesh having yield strength of 460 N/mm$^2$ at the mid-depth of the slab. The effects of shear slip on the deformation of the composite beams are not considered in the study, and full interaction between the concrete slab and the steel beam is assumed.

The unprotected secondary beams were designed for a load ratio of 0.45 at the fire limit state. The calculated critical temperatures of the protected secondary and primary beams were 588 °C and 592 °C respectively. A generic protection scheme based on the section factor was adopted, so that the protected beam temperatures were limited to 550 °C at 60 minutes. Details of the composite slab panel used in this study are given in Table 1.

### *4.1. Thermal analysis*

A thermal analysis was performed to ascertain the temperature distributions through the protected and unprotected beams and the concrete slab. Under exposure to the standard temperature-time curve, the simplified process from EN1993-1-2 [2] was used to calculate the temperature ($\theta$) distributions across the parts of the perforated steel section, as functions of their individual section factors, as shown in Fig. 5. A one-dimensional thermal analysis was performed on the concrete slab, using a thermal analysis program (FPRCBC-T), developed by Huang et al. [20]. This analysis generated the temperature distribution across an average slab effective thickness ($h_{eff}$) of 104.5 mm (104.5 mm = 130 mm-51 mm/2). Fig. 6 shows temperatures of the individual parts of unprotected steel sections and concrete slab when exposed to a standard fire curve.

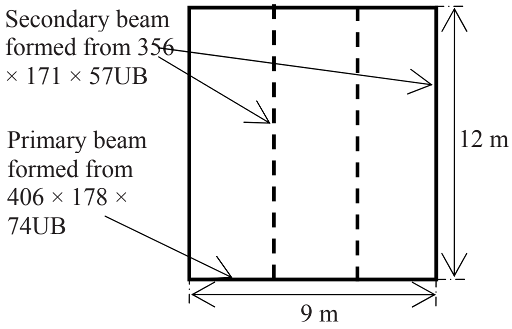


Fig. 3. Slab panel geometry.

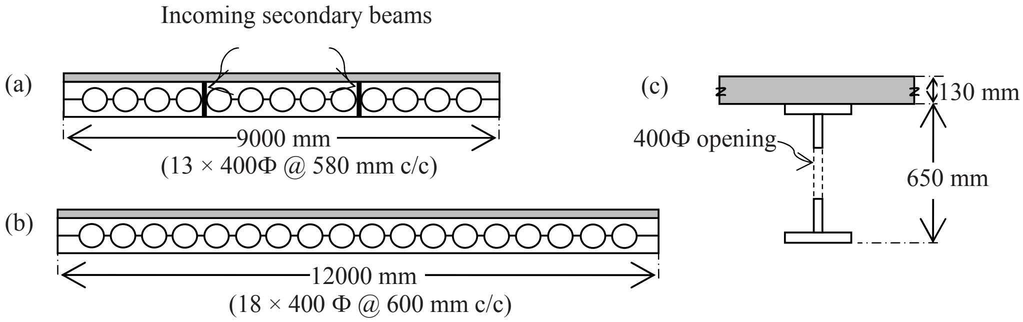


Fig. 4. Beam geometries (a) primary beam; (b) secondary beam and (c) typical cross-section.

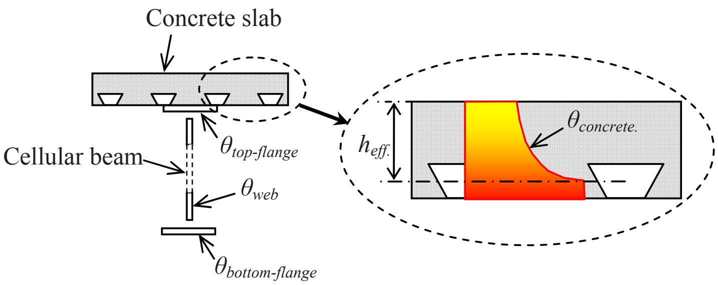


Fig. 5. Temperature distribution in steel section and concrete slab.

Table 1. Details of composite slab panel

| Primary and secondary beams | Opening | Web-post width | Spacing | Strength |
|---|---|---|---|---|
| PB: Top-tee: 406 × 178 × 74UB | 13 × 400 mm | 200 mm | 580 mm | 355 N/mm$^2$ |
| Bottom-tee: 406 × 178 × 74UB | | | | |
| SB: Top-tee: 356 × 171 × 57UB | 18 × 400 mm | 180 mm | 600 mm | 355 N/mm$^2$ |
| Bottom-tee: 356 × 171 × 57UB | | | | |
| Concrete slab: 9 m by 12 m, 130 mm thick, strength 35 N/mm$^2$ | | | | |
| A252 reinforcement mesh, yield strength 460 N/mm$^2$ | | | | |

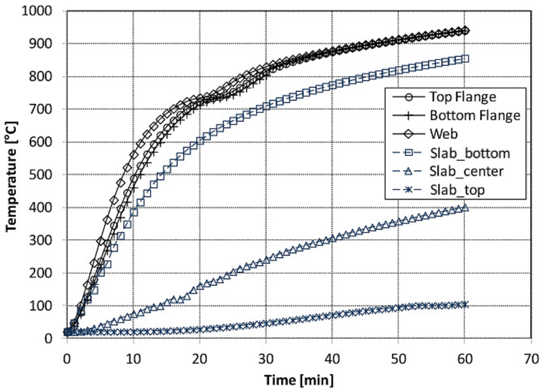


Fig. 6. Temperatures of the individual parts of the unprotected beam, and top, centre and bottom surface of the slab panel system.

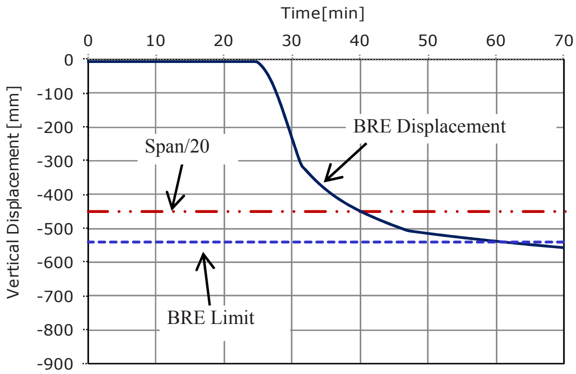


Fig. 7. Deflection limits for the slab panel system.

### 4.2. Structural analysis

Abu et al. [17] analyze a number of protection schemes and support conditions to examine the effects of edge support and reinforcement ratios on slab panel failure in fire. Based on the same analytical methodology, an investigation into the fire performance of cellular beams as part of composite slab systems is reported herein. The study on the 9 m × 12 m slab panel examined various support conditions intended to provide the necessary vertical support for tensile membrane action, in order to observe the difference in failure times which can be attributed solely to support conditions. For comparison, other analyses were performed with different edge conditions, to determine their effects on tensile membrane action. The finite element models are numbered as follows;

- S1: Generic protection and vertical support at corners
- S2: Generic protection with rigid vertical support along slab panel edges
- S3: Generic protection and rotational restraint on protected secondary beams bottom flange
- S4: The assumption of cold perimeter beams

The finite element analyses were compared with the BRE allowable vertical deflection limit for a fire resistance times up to 60 minutes, the required vertical deflection obtained from the BRE approach, and a limiting deflection of *short span/20* (450mm). The BRE allowable deflection limit curve was based on the assumptions of the BRE membrane action fire design method [14], taken into consideration the capacity of intermediate secondary beams with web-openings. The required vertical displacement for load capacity was obtained by the generic BRE Method using an A252 mesh within the concrete slab panel. The deflection limit of *short span/20* accords with the furnace testing code [21] and is commonly used in structural fire engineering design. Fig. 7 shows the vertical displacements used in the comparison.

### 4.3. Results and discussion

Figure 8 shows the deflection limits for the 9 m × 12 m slab panel analyses, compared against the finite element models S1 and S2. Panel S1 has its four corners supported, while model S2 is supported vertically along its perimeter beams. The Bailey-BRE method deems S2 to be adequate with A252 mesh, as shown. S1 is a more realistic representation of a slab panel in fire. However it exceeds the deflection criteria at 22 minutes, while model S2 just satisfies the allowable deflection limit at 60 minutes.

An investigation into the apparent failure of S1 is shown in Fig. 9. Displacements of the centre of the panel relative to the mid-points of the protected secondary beam (SB) and protected primary beam (PB) are plotted. It can be seen that the protected beams satisfied the allowable deflection limit at 60 minutes; however S1 suffered collapse of the unprotected intermediate secondary beams, caused by their loss of strength and stiffness as they approached the design temperature of 550 °C. This is emphasized in Fig. 10, which shows the deformation of the modelled quarter of the slab panel. From the figure, it can be seen that the supporting protected perimeter beams maintained their load-carrying capacity and were subject to small vertical displacements, although it can be observed that web-post buckling of the protected secondary beam occurred near to a support (Fig. 11). The increase of temperature-induced distortional buckling of the unprotected secondary beam, driving the bottom-tee section laterally out of its original plane, caused bending of the beam, at which point only the top-tee made a real contribution to support of the composite slab. This confirms the early failure of the slab panel, as the lateral deformation of the intermediate beam caused large deformation of the panel, and the BRE recommended deflection limit was no longer fulfilled.

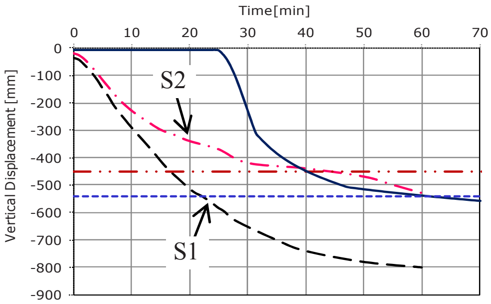


Fig. 8. Vertical displacement of slab panel with vertical support at corners and along slab panel edges.

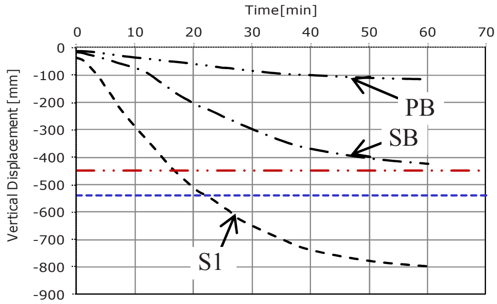


Fig. 9. Vertical displacement of slab panel and edge beams with vertical restraints at corners only.

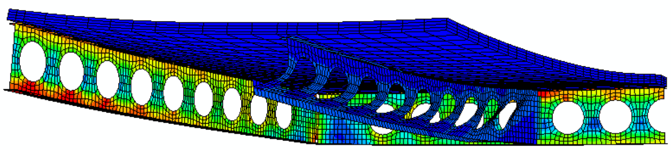

Fig. 10. Intermediate unprotected beam deformation.

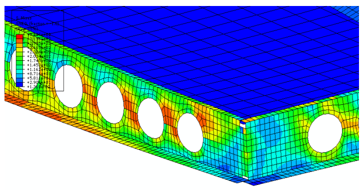

Fig. 11. Protected beam – web-post buckling.

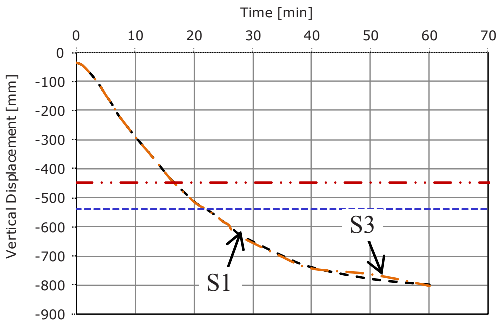


Fig. 12. Vertical displacement of slab panel with vertical support at corners and rotational restraint to protected secondary beam lower flange.

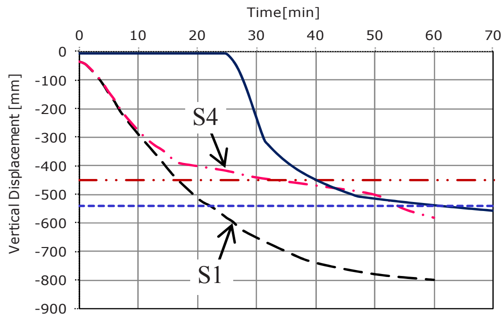


Fig. 13. Vertical displacement of slab panel with cold perimeter beams (20 °C), vertical support at corners and along slab panel edges.

The effect of rotational restraint to the bottom flanges of the protected secondary beams of the slab panels was also investigated. The results (Fig. 12) show that slab S3 behaved similarly to model S1, indicating that there is no lateral distortional buckling of the bottom tee of either protected secondary beam in the two models, and therefore no significant difference in deflection at the middle of the slab panels was recorded. Fig. 13 shows the results for the slab panel with cold perimeter beams (S4), indicating that the finite element model compared very well with the generic BRE required deflection, just crossing the allowable vertical displacement limit at about 55 minutes.

## 5. Conclusions

A number of support conditions have been analysed, and it is clear that considerable restraint is provided either by vertically supported corners or by continuous edges of slab panels. The results show that the BRE membrane action design method gives a good prediction of slab panel behaviour if the perimeter beams remain stiff. Practically, however, the support conditions where a slab panel is vertically restrained only at its four corners (model S1) gives more realistic results, and therefore the use of steel beams with web-openings to form part of the slab panel composite system, with an enhancement from tensile membrane action in fire, may need to be considered carefully. Based on the analyses, the intermediate unprotected beams failed in distortional buckling rather than the web-post buckling or Vierendeel bending which are the common failure modes observed in cellular beams.

The results show that the protected perimeter beams maintained their load carrying capacity and were subject only to small vertical displacements, although web-post buckling was observed to occur on the protected secondary beam near to a support. This suggests that maintenance of vertical support is not sufficient for a slab panel with cellular beams. Web-post buckling or the Vierendeel mechanism may govern the mode of structural failure, indicating that the sizes of openings and their positioning necessitate careful design. Although the study has shown that increasing the level of fire protection to the perimeter beams may be beneficial, the level to which these need to be protected requires further study.

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